Chapter 8
Reevaluation of Existing Dams

8-1. General

Existing gravity dams and foundations should be reevaluated for integrity, strength, and stability when:

a. It is evident that distress has occurred because of an accident, aging, or deterioration.

b. Design criteria have become more stringent.

c. Excavation is to be performed near existing structures.

d. Structural deficiencies have been detected.

e. Actual loadings are, or anticipated loadings will be, greater than those used in the original design. Loadings can increase as a result of changed operational procedures or operational deficiencies, an increase in dam height, or an increase in the maximum credible earthquake as a result of seismological investigations. Conditions such as excessive uplift pressures, unusual horizontal or vertical displacements, increased seepage through the concrete or foundation, and structural cracking are indications that a reevaluation should be performed.

8-2. Reevaluation

The reevaluation should be based on current design criteria and prevailing geological, structural, and hydrological conditions. If the investigations indicate a fundamental deficiency, then the initial effort should concentrate on restoring the dam to a safe and acceptable operating condition. Efforts could include measures to reduce excessive uplift pressures, reduce leak, repair cracks, or restore deteriorated concrete. Should restoration costs be unreasonable or should the fundamental deficiency be due to changes in load or stability criteria, a detailed analysis should be performed in accordance with the following procedures. The evaluation and repair of concrete structures is covered by EM 1110-2-2002. Reevaluation of structures not designed to current standards should be in accordance with the requirements of ER 1110-2-100.

8-3. Procedures

The following procedures shall be used in evaluating current structural conditions and determining the necessary measures for rehabilitation of existing concrete gravity dams.

a. Existing data. Collect and review all the available information for the structure including geologic and foundation data, design drawings, as-built drawings, periodic inspection reports, damage reports, repair and maintenance records, plans of previous modifications to the structure, measurements of movement, instrumentation data, and other pertinent information. Any unusual structural behavior that may be an indication of an unsafe condition or any factor that may contribute to the weakening of the structure's stability should be noted and investigated further.

b. Site inspection. Inspect and examine the existing structure and site conditions. Any significant difference in structure details and loading conditions between existing conditions and design plans and any major damage due to erosion, cavitation, undermining, corrosion, cracking, chemical reaction, or general deterioration should be identified and evaluated.

c. Preliminary analyses. Perform the preliminary analyses based on current structural criteria and available data. If the structure does not meet the current criteria, list the possible remedial schemes and prepare a preliminary cost estimate for each scheme. ER 1130-2-417 should be followed as applicable.

d. Design meeting. Schedule a meeting when the preliminary analyses indicate that the structure does not meet current criteria. The meeting should include representatives from the District, Division, CECW-E, and CECW-O to decide on a plan for proposed analyses, the extent of the sampling and testing program, the remedial schemes to be studied, and the proposed schedule. This meeting will facilitate the design effort and should obviate the need for major revisions or additional studies when the results are submitted for review and approval.

e. Parametric study. Perform a parametric study to determine the effect of each parameter on the structure's safety. The parameters to be studied should include, but not be limited to, unit weight of concrete, groundwater levels, uplift pressures, and shear strength parameters of rock fill material, rock foundation, and structure-foundation interface. The maximum variation of each parameter should be considered in determining its effect.

f. Field investigations. Develop an exploration, sampling, testing, and instrumentation program, if needed, to determine the magnitude and reasonable range of variation for the parameters that have significant effects on the safety of the structure as determined by the parametric study. The Division Material Laboratory should be used to the maximum extent practicable to perform the testing in accordance with ER 1110-1-8100.

g. Detailed structural analyses. Perform detailed analyses using data obtained from studies, field investigations, and procedures outlined in Chapters 4 and 5. Three-dimensional modeling should be used as appropriate to more accurately predict the structural behavior.

h. Refined structural analysis. The conventional methods described in Chapters 4 and 5 may be more conservative than necessary, especially when making a determination as to the need for remedial strengthening to improve the stability of an existing dam. If the conventional analyses indicate remedial strengthening is required, then a refined finite element analysis should be performed. This refined analysis should accurately model the strength and stiffness of the dam and foundation to determine the following:

(1) The extent of tensile cracking at the dam foundation interface.

(2) The base area in compression.

(3) The actual magnitude and distribution of foundation pressures.

(4) The magnitude and distribution of concrete stresses.

Information relative to refined stability analysis procedures can be found in Technical Report REMR-CS-120 (Eberling et al., in preparation).

i. Review and approval. Present the results of detailed structural analyses and cost estimates for remedial measures to the Division Office for review and approval. If a deviation from current structural criteria was made in the analyses, the results should be forwarded to CECW-ED for approval. The required basis for deviating from current structural criteria is given in paragraph 8-4*b*.

j. Plans and specifications. Develop design plans, specifications, and a cost estimate for proposed remedial measures in accordance with ER 1110-2-1200.

8-4. Considerations of Deviation from Structural Criteria

a. The purpose of incorporating a factor of safety in structural design is to provide a reserve capacity with respect to failure and to account for strength variability of the dam and foundation materials. The required margin depends on the consequences of failure and on the degree of uncertainties due to loading variations, analysis simplifications, design assumptions, variations in material strengths, variations in construction control, and other factors. For evaluation of existing structures, a higher degree of confidence may be achieved when the critical parameters can be determined accurately at the site. Therefore, deviation from the current structural criteria for an existing structure may be allowed under the conditions listed in paragraph 8-4*b*.

b. In addition to the detailed analyses and cost estimates as listed in paragraph 8-3*h*, the following information should also be presented with the request for a deviation from the current structural criteria:

(1) Past performance of the structure, including instrumentation data and a description of the structure condition such as cracking, spalling, displacements, etc.

(2) The anticipated remaining life of the structure.

(3) A description of consequences in case of failure.

c. Approval of the deviation depends upon the degree of confidence in the accuracy of design parameters determined in the field, the remaining life of the structure, and the potential adverse effect on lives, property, and services in case of failure.

8-5. Structural Requirements for Remedial Measure

When it is determined that remedial measures are required for the existing structure, they should be designed to meet the structural criteria of Chapter 4.

8-6. Methods of Improving Stability in Existing Structures

a. General. Several methods are available for improving the rotational and sliding stability of concrete gravity dams. In general, the methods can be categorized as those that reduce loadings, in particular uplift, or those that add stabilizing forces to the structure and increase overturning or shear-frictional resistance. Stressed foundation anchor systems are considered one of the most economical methods of increasing rotational and sliding resistance along the base of the dam. Foundation grouting and drainage may also be effective in reducing uplift, reducing foundation settlements and displacements, thereby increasing bearing capacity. Regrouting the foundation could adversely affect existing foundation drainage systems unless measures are taken to prevent plugging the drains; otherwise, drain redrilling will be required. Various methods of transferring load to more competent adjacent structures or foundation material through shear keys, buttresses, underpinning, etc., are also possible ways of improving stability.

b. Reducing uplift forces. In many instances, measured uplift pressures are substantially less than those used in the original design. These criteria limit drain efficiency to a maximum of 50 percent. Many designs are based on efficiencies less than 50 percent. Existing drainage systems can produce efficiencies of 75 percent or more if they extend through the most pervious layers of the foundation, if the elevation of the drainage gallery is at or near tailwater, and if the drains are closely spaced and effectively maintained. If measured uplift pressures are substantially less than design values, then parametric studies should determine what benefit it may have towards improving stability. Uplift pressures less than design allowables should be data from reliable instrumentation which assures that the measured uplift is indicative of pressures within the upper zones and along the entire foundation. Uplift pressures can be reduced by additional foundation grouting and re-establishing drains. Uplift may also be reduced by increasing the depth of existing drains, adding new drains, or rehabilitating existing drains by reaming and cleaning.

c. Prestressed anchors. Prestressed anchors with double corrosion protection may be used to stabilize existing concrete monoliths, but generally should not be used in the design of new concrete gravity dams. They are effective in improving sliding resistance, resultant location, and excessive foundation pressure. Anchors may be used to secure thrust blocks or stilling basins for the sole purpose of improving sliding stability. The anchor force required to stabilize a dam will depend largely on the orientation of the anchors. Anchors should be oriented for maximum efficiency subject to constraints of access, embedded features, galleries, and stress concentrations they induce in the dam. Analyses of tensile stresses under anchor heads should be made, and reinforcing should be provided as required. Tendon size, spacing, and embedment length should be based on the required anchor force, and should be provided the geotechnical engineer for determination of the required embedment length. Design, installation, and testing of anchors and anchorages should be guided by information in "Recommendations for Prestressed Rock and Soil Anchors" (Post-Tensioning Institute (PTI) 1985). Allowable bond stresses used to determine the length of embedment between grout and rocks are recommended to be one half of the ultimate bond stress determined by tests. The typical values of bond strength given in the above referenced PTI publication may be used in lieu of test values during design, but the design value should be verified by test before or during construction. The first three anchors installed and a minimum of 2 percent of the remaining anchors selected by the engineer should be performance tested. All other anchors must be proof tested upon installation in accordance with the PTI recommendations. Additionally, initial lift-off readings should be taken after the anchor is seated and before the jack is removed. Lift-off tests of random anchors selected by the engineer should be made 7 days after lock-off and prior to secondary grouting. Long-term monitoring of selected anchors using load cells and unbonded tendons should be employed where unusual conditions exist or the effort and expense can be justified by the importance of the structure. In addition to stability along the base of the dam, prestressed anchors may be required for deep-seated stability problems as discussed in the following paragraph. Non-prestressed anchors shall not be used to improve the stability of dams.

8-7. Stability on Deep-Seated Failure Planes

A knowledge of the rock structure of a foundation is crucial to a realistic stability analysis on deep-seated planes. If instability is to occur, it will take place along zones of weakness within the rock mass. A team effort between the geotechnical and structural engineers is important in evaluating the foundation and its significance to the design of the dam. Deep-seated sliding is of primary interest as it is the most common problem encountered. Significant foundation features are: rock surface joint patterns that admit water to potential deep-seated sliding planes; inclination of joints and fracturing that affect passive resistance; relative permeability of foundation materials that affect uplift; and discontinuities such as gouge zones and faulting which affect both strength and uplift along failure planes. Strength values for failure planes are required for design. As these values are often difficult to define with a high level of confidence, they should be described in terms of expected values and standard deviations. Analyses of resultant location and maximum bearing pressure will also be required. Criteria for these loading conditions will be the same as in Chapter 4 for the dam.

a. Method and assumptions. Stability on deep-seated planes is similar to methods described in Chapter 4 for the dam. Tensile strength within the foundation is neglected except where it can be demonstrated by exploration and testing. Vertical and near vertical joints are assumed to be fully pressurized by the pool to which they are exposed. Normally a pressurized vertical joint will be assumed to exist at or near the heel of the dam. Uplift on flat and inclined bedding planes will be dependent on their state of compression and the presence of drains passing through these planes as described for dams in Chapter 3. Passive resistance will be based on the rock conditions downstream of the dam. Adversely inclined joints, faults, rock fracturing, or damage from excavation by blasting will affect available passive resistance.

b. Anchor penetration. Required anchor penetration depends on the purpose of the anchor. Anchors provided to resist uplift of the heel must have sufficient penetration to develop the capacity of the anchors. Anchors provided to resist sliding must be fully developed below the lowest critical sliding plane. Critical sliding planes are those requiring anchors to meet minimum acceptable factors of safety against sliding.

c. Anchor resistance. The capacity of the anchor to resist uplift should be limited to the force that can be developed by the submerged weight of the rock engaged by the anchor. Rock engaged will either be shaped as cones or intersecting cones depending on the length and spacing of the anchors. The anchor force that can be developed should be based on the pullout resistance of a cone with an apex angle of 90 deg. Tensile stresses will occur in the anchorage zone of prestressed anchors. The possibility of foundation cracking as a result of these tensile stresses must be considered. It is possible that cracks in the foundation could open at the lower terminal points of the anchors and propagate downstream. To alleviate this potential problem, a sufficient weight of submerged rock should be engaged to resist the anchor force, and the anchor depths should be staggered.

8-8. Example Problem

The following example is a gated outlet structure for an earth fill dam. The existing gated spillway monoliths are deficient in sliding resistance along a weak seam in the foundation which daylights in the stilling basin. A cross-section of the spillway monoliths is shown in Figure 8-1. The spillway monoliths are founded at elevation 840 on moderately hard silty shale. A continuous soft, plastic clay shale seam approximately 1/2 inch in thickness exists at elevation 830. A free body diagram showing forces acting on the gravity structure and foundation above the weak seam is shown in Figure 8-2. Even though the foundation drains penetrate the potential sliding plane, the drains are assumed ineffective as they are insufficient to drain a thin clay seam. The sliding plane is in full compression, and uplift is assumed to vary uniformly from upper pool head to zero in the stilling basin. A drained shear strength of 20° 30′ has been assigned to this potential sliding surface, and a sliding factor of safety of 0.49 has been calculated for loading condition No. 2, i.e., pool to top of closed spillway gates. The tailwater is below the level of the sliding surface. A summary of loads and the resulting factor of safety for this critical loading condition is shown in Table 8-1. The design of anchors to provide a required factor of safety of 1.70 is summarized in Table 8-2. The anchors are located as shown in Figure 8-3. Details of the anchors are shown in Figure 8-4. The 45-deg angle for the anchors was selected to minimize drilling and to provide a large component of resisting force without creating a potential upstream sliding problem during low pools. Tips of anchors are staggered to avoid tensile stress concentrations in the foundation. The anchors are embedded below the lowest sliding plane requiring anchors to meet required safety factors. Reinforcement similar to that used in post-tensioned beams is provided under the bearing plates to resist the high tensile bursting stresses associated with large capacity anchors. The anchors were tensioned in the sequence shown in Figure 8-4 to avoid unacceptable stress concentrations in the concrete monoliths. The anchors were designed, installed, and tested in accordance with PTI (1985). The anchors are designed for a working load of 826 kips and were locked-off at 910 kips (i.e., working load plus 10 percent) to allow for calculated relaxation of the anchors, creep in the concrete structure, and consolidation of the foundation. Proof testing of all anchors to 80 percent of ultimate strength confirmed the adequacy of the anchors for a working load of 826 kips per anchor (approximately 60 percent of ultimate strength). Each anchor successfully passed a 14th day lift-off test, secondary grouting was accomplished, and anchor head recesses were filled with concrete to restore the spillway profile.

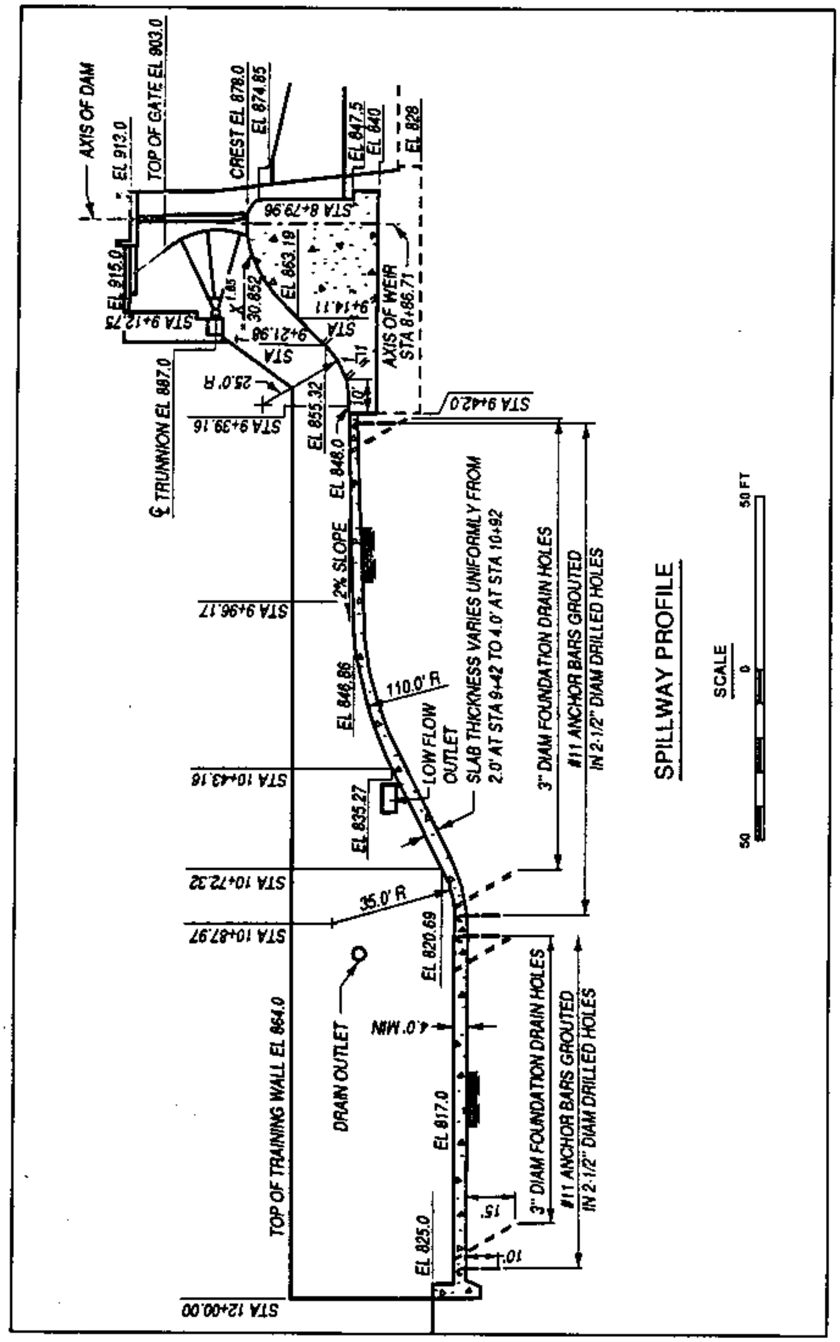

Figure 8-1. Cross-section of spillway monoliths

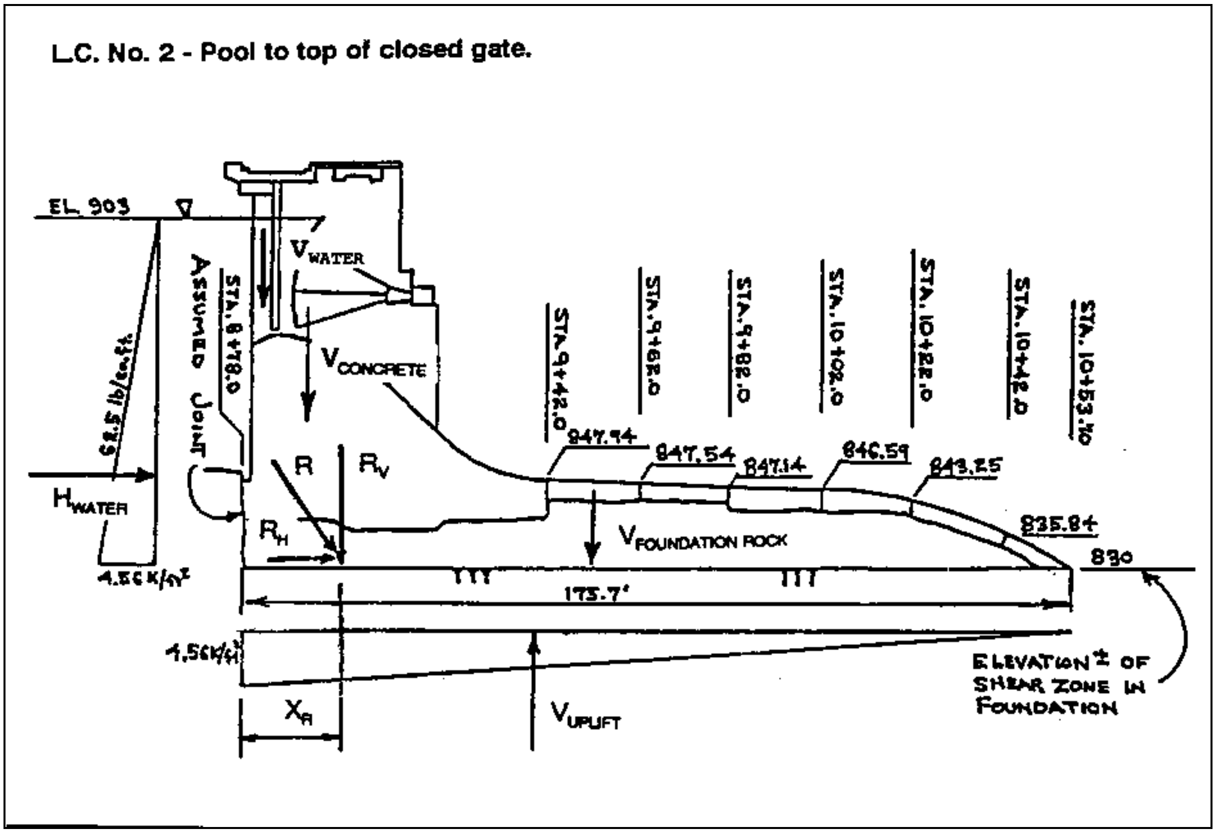

Figure 8-2. Free body diagram, R_y = resultant of vertical forces, R_H = resultant of horizontal forces, and X_R = distance from heel to resultant location on sliding plane

Table 8-1
Summary of Forces on the Sliding Plane. Loading Condition No. 2 (Pool at Top of Gates, Tailwater Below Sliding Surface)

	Σ Vert, kips	Σ Horz, kips
Concrete	11,910	
Rock (Saturated Weight)	13,160	
Machinery	10	
Gates	70	
Water Down	870	
Water Up	- 90	
Uplift	- 16,830	
Horizontal Water		6,990
Totals, Loading Condition No. 2	9,100	6,990

$$\textit{Sliding FS, Without Anchors} = \frac{TAN\ 20.5° \times 9{,}100}{6{,}990} = 0.49$$

Table 8-2
Summary of Forces on the Sliding Plane. Loading Condition No. 2, With Anchors

	Σ Vert, kips	Σ Horz, kips
Concrete	11,910	
Rock	13,160	
Machinery	10	
Gates	70	
Water Down	870	
Water Up	- 90	
Uplift	- 16,830	
Horizontal Water		6,990
Anchors (Vertical) 7 x 826 x 0.707	4,088	
Anchors (Horizontal)		- 4,088
Totals, Loading Condition No. 2	13,188	2,902

$$\textit{Sliding FS, With Anchors} = \frac{TAN\ 20.5° \times 13{,}188}{6{,}990 - 4{,}088} = 1.70$$

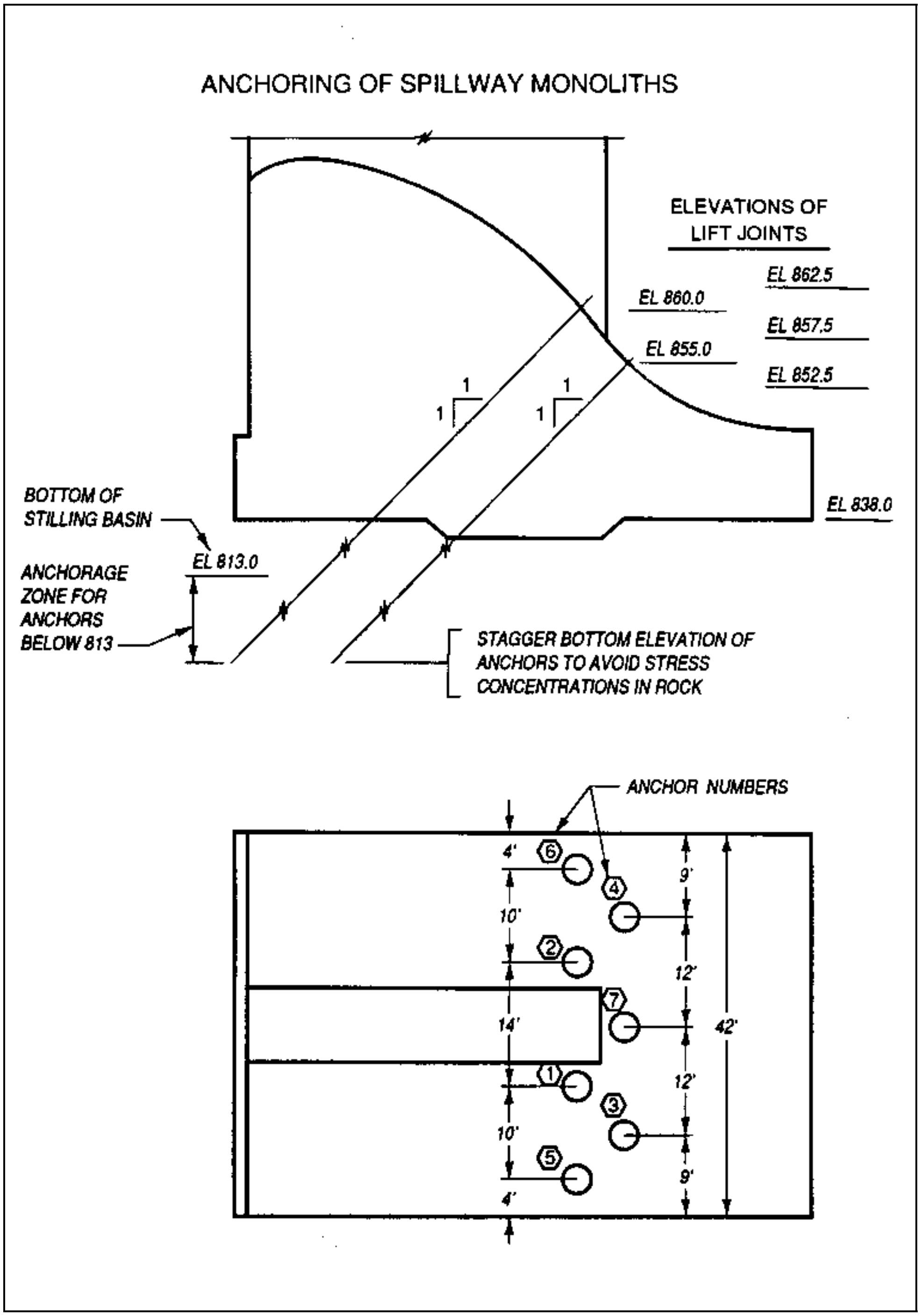

Figure 8-3. Location of anchors

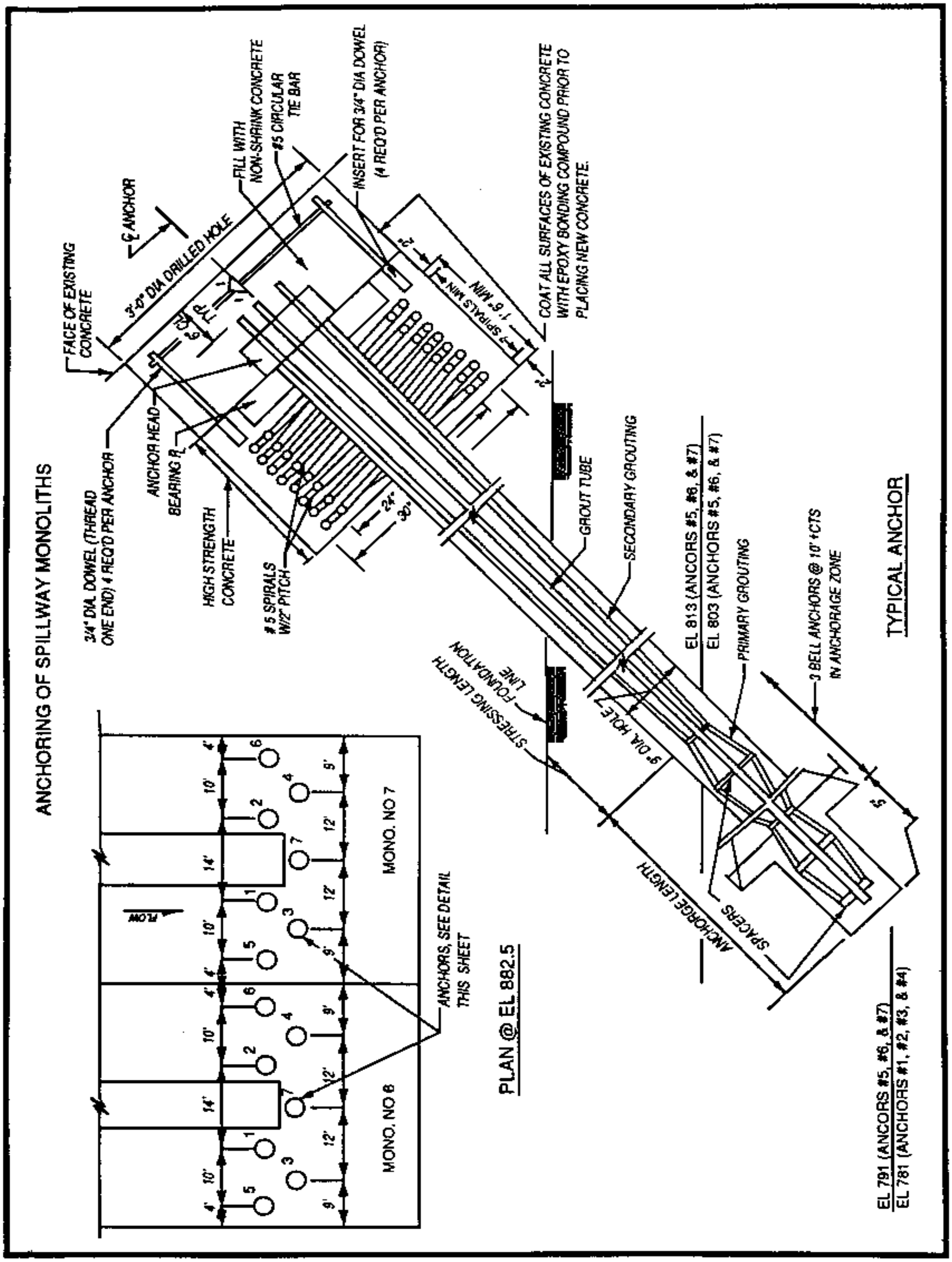

Figure 8-4. Details of anchors